Plywood Design Specification

Supplement No. 1

Design of Plywood Curved Panels

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Adopted by Plywood Fabricator Service Inc.

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Foreword

This publication presents the recommended method for the design of curved plywood roof panels spanning between load-bearing supports so that the stresses developed act circumferentially around the curve.

Fabrication requirements are given in FABRICATION OF PLYWOOD CURVED PANELS.

Working stresses and other design criteria are given in the PLYWOOD DESIGN SPECIFICATION, abbreviated PDS.

This recommended design method is based on standard engineering formulas, and was confirmed by tests recorded in American Plywood Association Laboratory Report No. 77, PLYWOOD ARCH PANELS. The Laboratory Report also contains a design example for a 16-ft span sandwich-type roof panel with a 2-ft 6-in. rise.

This method does not cover the special requirements for design of "vaults", which span parallel to the axis of the arch, and produce stresses at right angles to those in simply supported arches. Nor does it cover design of inverted curved panels, which require special consideration in design for concentrated loads. Such inverted panels are probably best designed as skins supported on curved beams.

Technical Services Division
American Plywood Association

A Word on Components

Plywood-lumber components are major structural members which depend on the glued joints to integrate the separate pieces into an efficient unit capable of carrying high stresses. Materials in these components may be stressed to an appreciably higher level than in conventional construction.

Since improperly designed or fabricated components could constitute a hazard to life and property, it is absolutely essential that not only the design method offered here be followed but that adequate quality control be maintained during manufacture.

To be sure that such control has been carefully maintained you may wish to contact the Plywood Fabricator Service (PFS). A requirement that each unit bear the PFS "Quality-Inspected" trademark will assure you of adequate independent inspection and testing of production.

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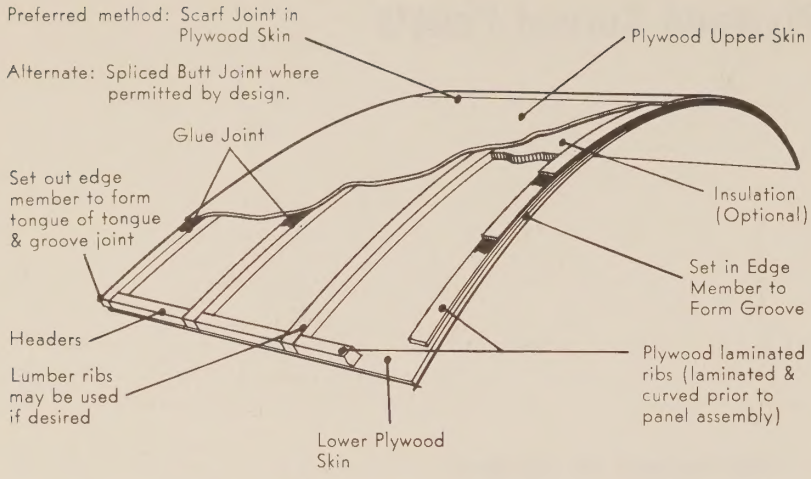
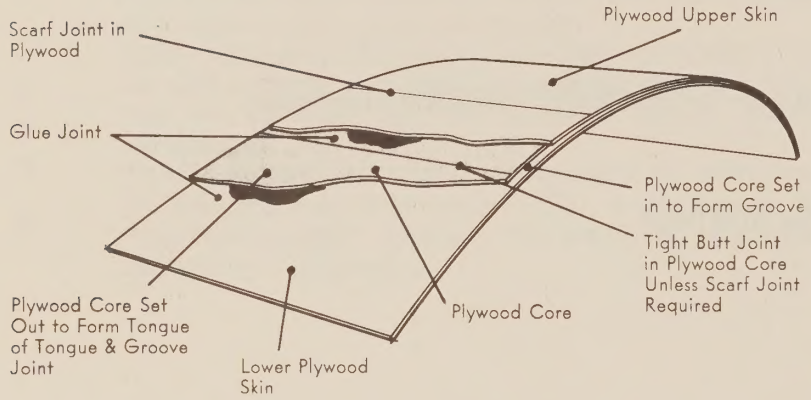

Preferred method: Scarf Joint in Plywood Skin
Alternate: Spliced Butt Joint where permitted by design.
Plywood Upper Skin
Glue Joint
Set out edge member to form tongue of tongue & groove joint
Insulation (Optional)
Set in Edge Member to Form Groove
Headers
Lumber ribs may be used if desired
Plywood laminated ribs (laminated & curved prior to panel assembly)
Lower Plywood Skin

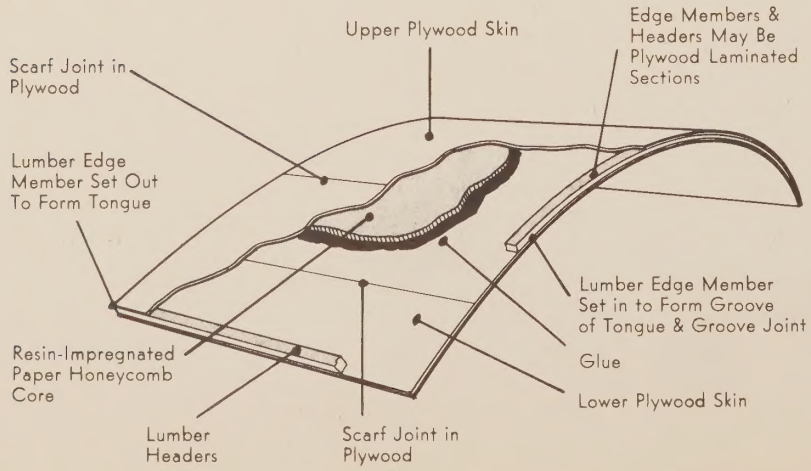
Typical Panel Using

Curved Plywood Ribs


Scarf Joint in Plywood
Plywood Upper Skin
Glue Joint
Plywood Core Set in to Form Groove
Tight Butt Joint in Plywood Core Unless Scarf Joint Required
Plywood Core Set Out to Form Tongue of Tongue & Groove Joint
Plywood Core
Lower Plywood Skin

Typical Panel Using

Solid Plywood Core


Upper Plywood Skin
Edge Members & Headers May Be Plywood Laminated Sections
Scarf Joint in Plywood
Lumber Edge Member Set Out To Form Tongue
Lumber Edge Member Set in to Form Groove of Tongue & Groove Joint
Glue
Resin-Impregnated Paper Honeycomb Core
Lower Plywood Skin
Lumber Headers
Scarf Joint in Plywood

Typical Panel Using

Lightweight Sandwich Core

Design of Plywood Curved Panels—PDS Supplement No. 1*

1. Types of Curved Panels

1.1 Structural Types

Curved panels may be designed as either of two structural types, depending on whether horizontal thrust is developed.

1.1.1 Curved Flexural Panels

This type performs in flexure in the same way as a conventional flat panel, acting as a simple beam, without developing horizontal thrust. To avoid damage to the supporting structure, bearing details must be designed accordingly, with provision for horizontal deflection. No tie rods are required.

Generally, thickness is greater than for arch panels of the same span. Construction is usually with spaced ribs, to achieve the necessary thickness. Stresses are only slightly affected by radius of curvature.

1.1.2 Arch Panels

Arch panels are stressed both in compression and in flexure. They exert horizontal thrust at the supports, and therefore require tie rods or abutments. These panels are relatively thin, and stresses are considerably affected by radius of curvature.

Arches are generally of constant cross section, and continuous from support to support, without fixity at the supports. In this case they are called "two-hinged", and are statically indeterminate.

For longer spans, two arch segments may be joined together to form a "three-hinged" arch. The joint, or hinge, is not moment-resisting, and the resulting member is statically determinate.

1.2 Shape

1.2.1 General

Curved panels may be made in any desired shape within the limits of fabrication of the materials used. No compound curvature (simultaneous curvature in two directions as for a dome shape) is possible with plywood. For convenience in design and fabrication, circular curves are generally specified.

1.2.2 Limits of Curvature

Panels of medium curvature (between perhaps 3-1 and 8-1 span-rise ratios) are most practical. With panels of low rise, outward thrusts become very high and small outward movements at the supports can cause undesirable deflections of the panel. Panels with very high rise must be especially designed for resistance to lateral forces such as wind, since they become less stable as they approach semi-circular shape.

1.2.3 Bending Radii

See Fabrication Specification for minimum bending radii of plywood skins.

1.3 Core Types

Plywood curved panels consist of full-length plywood skins top and bottom, spaced by and joined with glue to a structural core capable of resisting the shearing forces.

1.3.1 Spaced Ribs

For all spans the core may consist of single-piece or laminated plywood or lumber ribs spaced as required, either pre-glued or glued during panel assembly. Such construction can readily include blanket insulation. The ribs usually parallel the span, where they contribute to the moment of inertia of the panel, and serve to resist shear. For some short spans, however, the ribs may be oriented parallel to the axis of the arch, where they serve primarily as spacers for the plywood skins.

1.3.2 Plywood Core

For short spans the core may consist of one or more layers of plywood, butt-jointed or full length, usually glued over the full area to the skins. In some cases glue need be applied only in strips, making the attachment similar to that in spaced-rib panels. Such "strip-gluing", where applicable, offers reduced fabrication costs.

1.3.3 Sandwich Core

Sandwich material, such as resin-impregnated paper honeycomb or foamed plastic, may be used for the core where light weight is vital.

*Previously Design Method No. CP-8D

2. Design Considerations

2.1 Design Loads

2.1.1 Uniform Load

The design live loads must not be less than required by the governing building regulations. Dead load is the actual weight of the member and the elements it supports.

Curved panels must support dead load over the full area of the panel, plus live load over any portion of the panel. Loads may be assumed distributed over the horizontal projection of the panel.

For curved flexural panels the critical loading condition will be with dead load and live load distributed over the full panel area. For arch panels, both this condition, and that of live load over only one half of the span, should be considered.

2.1.2 Concentrated Erection Load

Curved panels should be adequate to support, at any point on the span, a concentrated load such as that of several workmen standing, or roofing materials piled in one area. This load might come before or after adjoining panels are fastened together.

Curved panels should be designed for a minimum concentrated erection load of 150 lb per ft of panel width, with a 25% increase in allowable stresses.

2.1.3 Wind Load

Panels, including anchorage details, must be designed to resist suction due to wind load, combined with internal pressure developed by wind through openings in the side walls, minus the dead load of the panels and roofing. Such loading may equal or exceed the gravity load, thereby causing stress reversal in the panels and tie rods.

2.2 Allowable Working Stresses

2.2.1 Plywood Faces

Stresses for design of plywood faces depend on the radius, the type of core, and the joints in the plywood.

2.2.1.1 Curvature Factor—For the curved portion of members the allowable unit stress in bending and in tension and compression parallel with the grain shall be modified through multiplication by the following curvature factor:

$$1 - 2000\left(\frac{t}{R}\right)^2$$

in which

t = distance between extreme fibers of plies parallel with the stress for one lamination (inches)

R = radius of curvature of a lamination (inches)

$\frac{t}{R}$ shall not exceed $\frac{1}{125}$.

2.2.1.2 With Spaced Ribs—Allowable stresses in compression and tension given in PLYWOOD DESIGN SPECIFICATION Section 3.14, and reduced for curvature as above may require further reduction. As described in DESIGN OF FLAT PLYWOOD STRESSED-SKIN PANELS, the ratio of actual rib spacing to design basic spacing for a given skin thickness will determine the need and extent of the reduction.

2.2.1.3 With Honeycomb, Foam, or Plywood Core—Working stresses in flexure, compression, and tension are given in PDS Section 3.14, and must be reduced for curvature as above.

2.2.2 Core Materials

The working stress in shear for paper honeycomb or foamed plastic core should not exceed one third of its ultimate shear strength in the appropriate direction for the moisture condition and type of end joint used.

2.3 Effective Sections

All plywood and all lumber having its grain parallel to the direction of stress may be considered effective in resisting the stress, subject to the following restrictions.

2.3.1 For Bending and Compression

2.3.1.1 Spaced-Rib Panels—See Section 1.4 of DESIGN OF FLAT PLYWOOD STRESSED-SKIN PANELS for reductions due to splice plates, and Section 2.5 for reductions due to transverse deformation.

2.3.1.2 Plywood Core and Laminated Ribs—The effective strength of solid-plywood-core panels and

of laminated ribs in a spaced-rib panel at a section containing a butt joint in any lamination may be determined by ignoring the butted lamination, and any other lamination containing a butt joint closer than 50 times the lamination thickness. When the butt joint occurs in the tension portion of the cross section, the allowable fiber stress in bending for the remaining parallel-grain material must be multiplied by 0.8.

2.3.1.3 Honeycomb or Foamed Plastic Core—Paper honeycomb or foamed plastic core should not be assumed to resist any flexural or direct stresses.

2.3.2 For Shear and Deflection

Shear and deflection of a curved panel containing butt joints in either the core or the faces may be based on the gross section of all material having its grain parallel with the direction of principal stress.

2.4 Allowable Deflection

The maximum deflection should not exceed 1/240th of the span under live load, nor 1/180th of the span under dead-plus-live load. There need be no limit for deflection under the concentrated erection load.

3. Design Method

3.1 Considerations Common to Both Curved Flexural Panels and Arch Panels

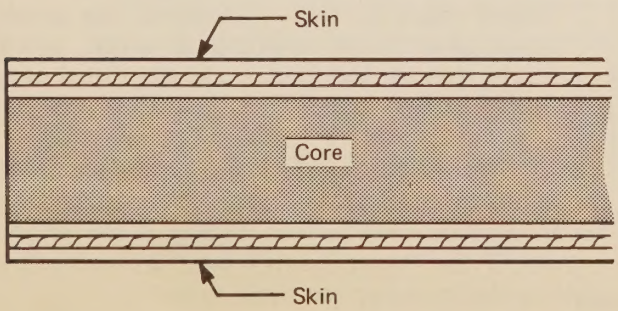
To prevent damage to the supporting structure, curved flexural panels must be designed so that no thrust can be developed. For arch panels, direct compression stress must be added to bending stress.

3.1.1 Spaced-Rib Panels

Design of curved spaced-rib panels is similar to that of flat stressed-skin panels as given in DESIGN OF FLAT PLYWOOD STRESSED-SKIN PANELS. The information in that design method is basic to all panel design.

3.1.2 Sandwich Panels

Design of curved sandwich panels is similar to that for flat sandwich panels explained in American Plywood Association Laboratory Report No. 93, LOAD-BEARING PLYWOOD SANDWICH PANELS. It is based on the assumption that parallel-grain material in the skins carries all bending forces, while the core carries only shear.



w = Uniform loading (psf)

P = Axial load (plf)

E = Modulus of elasticity of plywood (psi)

E_c = Modulus of elasticity of core (psi)

G_c = Modulus of rigidity of core (psi)

C_{cr} = Critical skin buckling stress (psi).

3.1.2.1 Basic Properties—Compute neutral axis, moment of inertia, and section modulus based on area of the skins. Figure two values of section modulus, one for top, one for bottom, if neutral axis is not at centerline of panel depth.

3.1.2.2 Critical Stress—Determine approximate skin buckling stress. Maximum combined stress, when computed, must be less than F_c, and less than one third of this value of C_{cr}.

$$C_{cr} = 0.5 \sqrt[3]{EE_cG_c}$$

This critical stress represents the value at which the skin may be expected to suffer local buckling, crushing the core or causing the core to fail in tension normal to the skins.

3.1.3 Radial Stresses

When a curved member is subjected to bending moment, a radial stress is induced. When the

moment is in the direction tending to decrease curvature (increase the radius) the stress is in tension. When the moment is in the direction tending to increase curvature (decrease the radius) the stress is in compression.

This radial stress is given by the following equation. It must be less than the allowable values given in Section 2.2 for plywood or lumber. For sandwich panel core material it should be less than one third of the allowable shear value of the core.

$$f_r = \frac{3}{2}\frac{M}{Rbh}$$

where

f_r = radial stress (psi) due to moment, M

M = bending moment (in.-lb) on a width of panel equal to the rib spacing for spaced-rib panels, or a 1-ft width for sandwich panels

R = radius of curvature at centerline of member (in.)

b = width of rib (in.) for spaced-rib panels, or 12" for sandwich panels

h = over-all depth of panel (in.).

3.1.4 Curvature Factor

Allowable stresses must be reduced for curvature, as in Section 2.2.1.

3.2 Curved Flexural Panels

3.2.1 Horizontal Deflection at Supports

The horizontal deflection at the free support of a curved panel that is free to deflect horizontally at one end may be determined from the following equation:

$$\Delta_H = \frac{4y\,\Delta_V}{\ell}$$

where

Δ_H = Horizontal deflection at support (in.)

y = Rise of curved panel at mid-span (in.)

Δ_V = Vertical deflection of curved panel at mid-span due to vertical load (in.)

ℓ = Horizontal span length (in.).

This equation, while approximate, is sufficiently accurate to determine the amount of space to allow at supports for the horizontal deflection to take place, so as to avoid developing thrust that is not provided for in the design.

3.2.2 Connections

3.2.2.1 Connections to Supports—Connections of panels to supporting members must provide for the horizontal movement calculated under Section 3.2.1 above. Lag screws or bolts in slotted or oversize holes in the panels are generally suitable. Connections must be adequate to resist the uplift as found in Section 2.1.3, as well as any shear that may be developed by diaphragm action.

3.2.2.2 Connections between Panels—Connections between panels must be adequate to make them perform as a unit; to transfer concentrated erection loads between panels without excessive differential deflection; and to transfer any other shear developed by diaphragm action. Tongue-and-grooved joints or nailed plywood battens are generally suitable.

3.3 Arch Panels

(See Appendix for Outline Design Example.)

3.3.1 Design

Two-hinged arch panels may be designed using the procedure outlined below. The solution for three-hinged arches is the same as that outlined for two-hinged arches, except that the horizontal thrust is calculated by statics instead of by use of the graph, Figure 3.3.1.1.

3.3.1.1 Reactions—Calculate the vertical reactions and horizontal thrusts of the arch for the various conditions of loading. (See Section 2.1.) Figure 3.3.1.1 will simplify this step.

> Since the Thrust Coefficient graph is set up for loads at the tenth points of the span, it is necessary to break up the uniform load into equivalent concentrated loads acting at these points. Ignore the load on 0.05 of the span adjacent to each support (it has negligible influence on the thrust) and divide the remainder into equal sections with concentrated loads at their midpoints. The graph then gives thrust coefficients, which, when multiplied by the load, yield the thrust produced by that load.

3.3.1.2 Moments—Calculate by statics and plot the moments at convenient intervals along the arch for the various conditions of loading. A convenient interval, if the Thrust Graph is used, is obtained by dividing the span into ten equal parts.

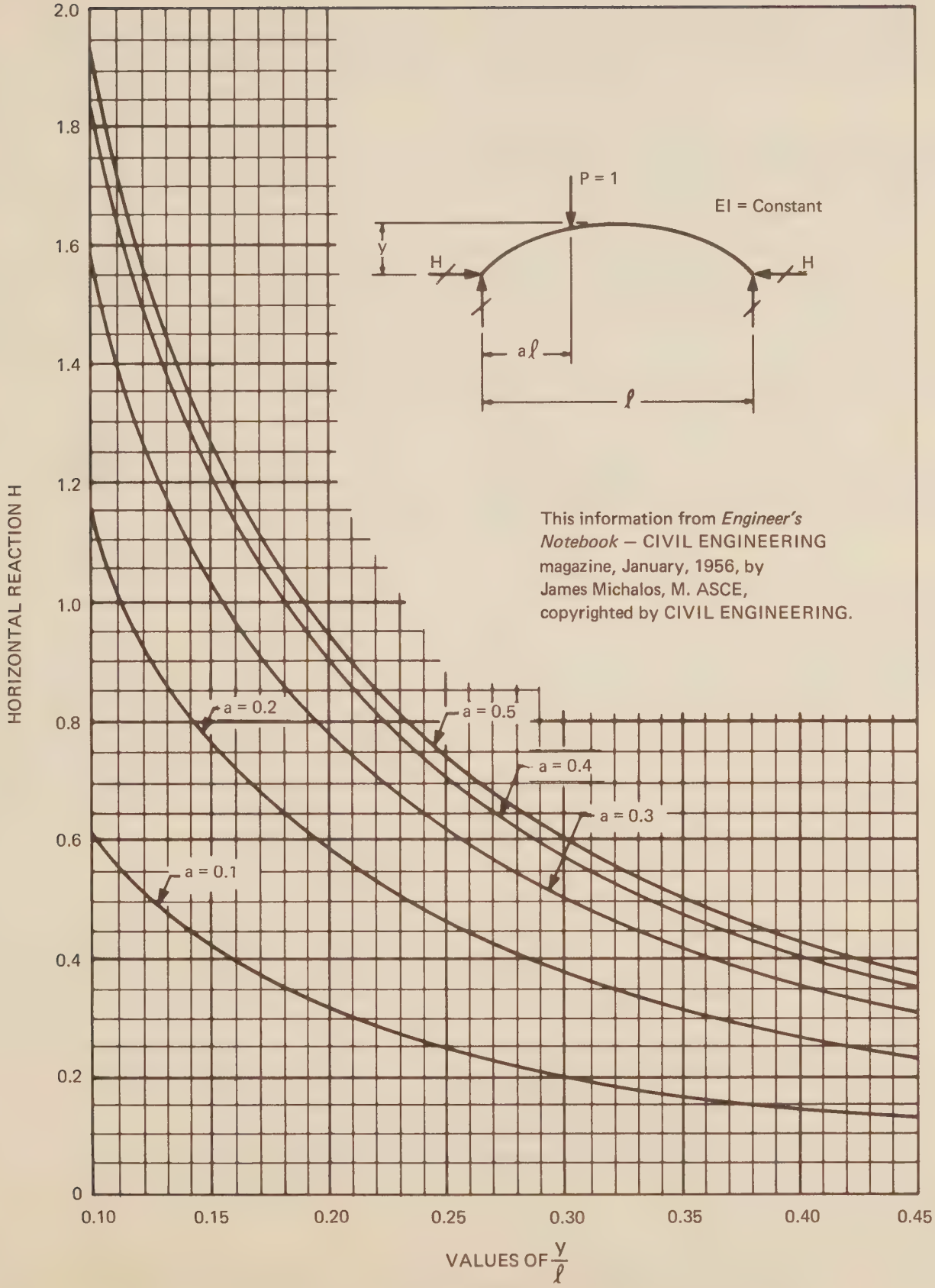


FIGURE 3.3.1.1 Thrust Coefficients for 2-Hinged Circular Arches

3.3.1.3 Maximum Moments—By inspection of the moment curves, determine the most severe loading condition and the magnitude of maximum moment. Worst case for maximum moment will usually occur either with dead load plus concentrated erection load, or with dead load plus half-span live load. In either case it will be near the quarter point of the span.

3.3.1.4 Direct Stress—Calculate the direct stress at the point of maximum moment by resolving acting forces into components tangential to the slope of the arch.

3.3.1.5 Trial Section—Guided by the moment and direct stress figured above, assume a trial cross section for the arch and calculate its weight and section properties.

3.3.1.6 Combined Stresses—Calculate the maximum combined stresses in the plywood at the point of maximum moment.

The combined stresses due to bending and compression must not exceed values given in Section 2.2; they may be calculated by the formula:

$$f = \frac{P}{A} \pm \frac{M}{S}$$

where

f = Stress in extreme fiber (lb per sq in.) tension or compression

P = Direct force (lb per ft width of arch)

A = Area of arch cross section (sq in. per ft width of arch)

M = Bending moment (in.-lb)

S = Section modulus (in.^{3} per ft width of arch).

3.3.1.7 Shear—Determine the maximum shear normal to the slope of the arch. Maximum shear occurs at the ends for both dead load and full-span uniform live load. Maximum shear for half-span load occurs at the center of the span, with shears at the ends almost as high for most span-to-rise ratios. (This shear equals one quarter of the unbalanced load.) Maximum shear due to concentrated erection load will probably occur at an end, when the load is placed near that end.

3.3.1.7.1 Spaced-Rib and Plywood-Core Panels—The shear stress must not exceed rolling shear values for plywood given in the PLYWOOD DESIGN SPECIFICATION; it may be calculated by the formula:

$$f = \frac{VQ}{Ib}$$

where

f = Shear stress (lb per sq in.) either rolling shear in plywood, or horizontal shear in rib

V = Shear acting normal to the slope of the arch (lb per foot of arch width, or per rib section)

Q = First moment about neutral axis of area of parallel-grain material from panel face inward to plane at which shear stress is to be calculated (in.^{3} per ft width of arch panel, or per rib section)

I = Moment of inertia of arch panel (in.^{4} per ft width of arch panel or per rib section)

b = Width of rib (in.) for spaced-rib panel; width of glued strip, for strip-glued solid-core panel; or 12" for solid-plywood-core panel glued over its total area.

3.3.1.7.2 Honeycomb or Foamed-Plastic Core—Shear stress in the core shall not exceed values given in Section 2.2.2; stress may be computed by the following formula (which allows credit for that portion of the shear resisted by wood edge members):

$$q = \frac{2V_2}{(h + c)b}$$

where

q = Shear stress in core (lb per sq in.)

$V_2 = V\text{-}V_1$ = Total shear acting normal to the slope of the arch minus allowable shear carried by wood framing members (lb per panel width)

h = Total depth of panel (in.)

c = Thickness of core (in.)

b = Width of core per panel (in.).

3.3.1.8 Radial Stresses—Calculate radial stresses as in Section 3.1.3.

3.3.1.9 Deflection—Calculate the maximum deflection of the arch for the various conditions of loading.

Deflection of arch panels must not exceed values given in Section 2.4, and may be determined by solving the following equation, which ignores the minor portions of deflection due to shear and direct stress:

$$\Delta_a = \frac{s}{EI} \Sigma Mm$$

where:

Δ_a = Deflection at point "a" in same direction as unit load (in.)

M = Moment due to the load or loads which cause the deflection (in.-lb per ft of panel width)

m = Moment due to unit load at point "a" (unit load-in.)

s = Length (in.) of segments along arch. (Ten equal-length segments are usually sufficient, though any number may be used.)

E = Modulus of elasticity (lb per sq in.) of plywood faces in arch (1,800,000 for Group 1)

I = Moment of inertia of arch panel ($in.^4$ per ft of panel width).

3.3.1.10 Final Section—Revise the trial cross section if necessary and repeat appropriate calculations.

3.3.2 Connections and Supports

3.3.2.1 Connections to Supports—Connections of panels to supporting members may be with nails, lag screws, bolts, or other means adequate to resist the maximum horizontal thrust and uplift, as well as any shear developed by diaphragm action.

3.3.2.2 Connections Between Panels—Connections between panels must be adequate to make them perform as a unit; to transfer concentrated erection loads between panels without excessive differential deflection; and to transfer any shear developed by diaphragm action. Tongue-and-grooved joints, or nailed plywood battens are generally suitable.

3.3.2.3 Supports—Horizontal supporting members for panels must be adequate to resist vertical and horizontal loads between their own supports without excessive deflection. Horizontal thrust must be resisted by properly designed tie rods, struts, or abutments.

3.3.2.4 Tie Rods—Where tie rods are used to resist thrust, such rods or other effective means of resisting thrust must be placed in all bays.

3.3.2.5 Abutments—Where abutments are used at the outer edges of exterior bays, means of resisting thrust must be placed in all bays. Such means of resisting thrust may include struts, shear walls, or other devices. Spacing should be as required by the interior panel supporting members. Such struts are required to provide for unbalanced loads when not all bays are equally loaded.

3.3.2.6 Diaphragms—Where diaphragm action is assumed, struts and chords must be provided as required by diaphragm forces.

Appendix – Outline Design Example for Arch Panel

Problem: Design curved arch-type panel with 8-ft span, 6-ft radius of curvature, for live load–32 psf.

Assume dead load–8 psf; conc. erection load–150 plf. (Apply at 1/4 point.)

A1. Determine External and Internal Forces

A1.1 Distances

Break arch into ten segments and compute vertical and horizontal distances to each.

Point	x	y
0	0	0
.1	0.8′	0.60′
.2	1.6′	1.03′
.3	2.4′	1.31′
.4	3.2′	1.47′
.5	4.0′	1.53′
.6	4.8′	1.47′
.7	5.6′	1.31′
	etc.	

A1.2 Thrusts

From Figure 3.3.1.1, page 5, determine thrust coefficient, H, for each increment, and for total uniform load. (H for half-span uniform load is just one half H for total uniform load.) Maximum thrust will occur with full live load plus dead load on the total span.

Load at Point	Thrust (coefficient)	Thrust, H equals coefficient x w w = 0.8 ft x 40 psf
0	0	0
.1	0.32	10.2
.2	0.60	19.2
.3	0.81	25.9
.4	0.95	30.4
.5	0.99	31.7
.6	0.95	30.4
.7	0.81	25.9
	etc.	
All points	6.35	203.2

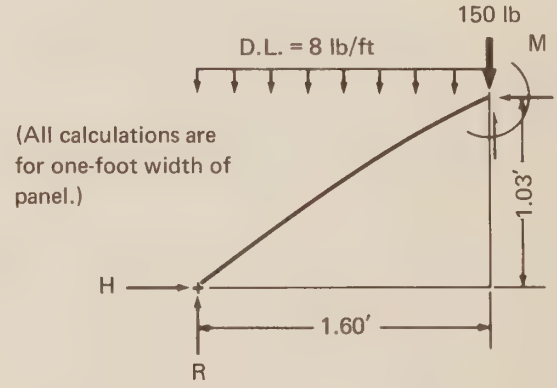
ΣH (for full live load plus dead load)–203 lb per ft of width.

A1.3 Vertical Reaction

Compute maximum vertical reaction. It will be due to full live load plus dead load on the total span.

A1.4 Maximum Moment

By simple statics find maximum moment at each point. Worst case for maximum moment will usually occur either with dead load plus concentrated erection load, or with dead load plus half-span live load. In either case it will occur near the quarter point of the span. In this example, concentrated load causes by far the highest moment.



$$R = \frac{6.4}{8} \times 150 + \frac{8 \times 8}{2}$$

$$= 152 \text{ lb per foot of width}$$

$$H = 0.60 \times 150 + 6.35 \times 8 \times 0.80$$

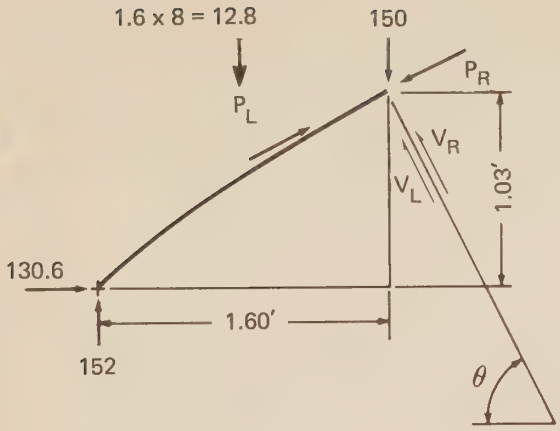
$$= 130.6 \text{ lb per foot of width}$$

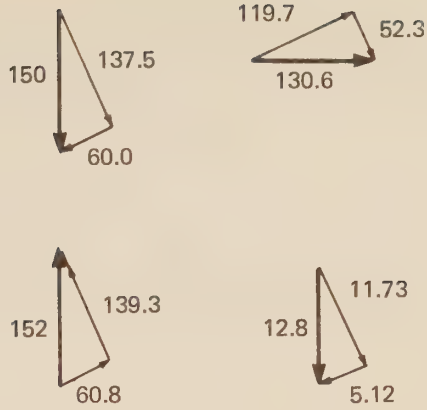
$$M = 152 \times 1.6 - 130.6 \times 1.03 - \frac{8 \times (1.6)^2}{2}$$

$$= 98.4 \text{ lb/ft per foot of width}$$

A1.5 Internal Forces

By statics, compute internal "forces" tangent and perpendicular to the slope of the arch at the point of maximum moment. This shear and "direct load" are those due to the applied load which causes the maximum moment—in this case the concentrated erection load.





P_R = -60.0 + 119.7 + 60.8 - 5.12 = 115 lb per foot

P_L = 115 + 60 = 175 lb per foot

V_R = 137.5 + 52.3 - 139.3 + 11.73 = 62 lb per foot

V_L = 52.3 - 139.3 + 11.7 = -75 lb per foot

A1.6 Summary of Results

H max (for full LL + DL over entire span) ..... 203 plf

Max vertical end reaction (from full load on entire span) ..... 160 plf

M max (for concentrated erection load at 0.2ℓ) ..... 98 ft-lb/ft

Shear at 0.2ℓ (for concentrated erection load at 0.2ℓ) ..... 75 lb-/ft

Direct axial load at 0.2ℓ (for concentrated erection load) ..... 175 lb/ft

M (from DL + half-span LL) ..... 34 ft-lb/ft

Shear at 0.5ℓ (for erection load at 0.5ℓ) ..... 75 lb/ft

Shear at end (for erection load near end) ..... 109 lb/ft

A2. Calculate Member Sizes To Resist Above Forces

A2.1 Stressed-Skin Panels

See DESIGN OF FLAT PLYWOOD STRESSED-SKIN PANELS for design of stressed-skin-type curved panels.

A2.2 Sandwich Panels

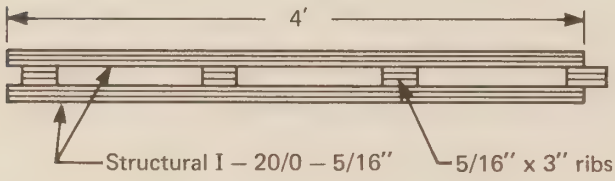
See Laboratory Report No. 93, LOAD-BEARING PLYWOOD SANDWICH PANELS, for design of sandwich-type curved panels. (For quick reference by those familiar with the basic principles of such designs, the method is summarized in this design method, DESIGN OF PLYWOOD CURVED PANELS, Section 3.1.2.)

A2.3 Curvature

In either case, take into account stress reduction for curvature, as covered in Section 3.1.3.

A2.4 Trial Section

Trial section, using DESIGN OF FLAT PLYWOOD STRESSED-SKIN PANELS



A_{II} = 21.6 $in.^2$ per 4-ft width

I = 2.11 $in.^4$ per 4-ft width

Q_{NA} = 3.12 $in.^3$ per 4-ft width

A2.5 Allowable Stresses

Ratio $\frac{\text{clear distance}}{\text{"b"}} > 1$; factor = 67%

Curvature factor = 0.962

Then allowable stress

$F_c = 1550 \times 1.15 \times 0.67 \times 0.962 = 1149$ psi

Basic

Normal to Snow

Rib Spacing Factor

Curvature Factor

A2.6 Final Stresses on Trial Section

Maximum combined stress, $f_{max} = \frac{P}{A} \pm \frac{Mc}{I}$

for worst case (concentrated erection load)

$$f_{max} = \frac{P}{A} + \frac{Mc}{I}$$

$$= \frac{4 \times 175}{21.6} + \frac{4 \times 98 \times 12 \times 15/32}{2.11}$$

$$= 31.1 + 993$$

$$= 1078 \text{ psi} < 1149 \text{ psi}$$

f_{max} = 1078 psi

Radial stress = 11.6 psi

Horizontal shear = 53.7 psi

(Note that with plywood stringers, this stress must be resisted by the plywood in "rolling shear".)

Tie rods required - could use 1/2"ϕ at 12'-0"

Trial Section O.K. as shown.

Notes:

Notes:

AVERY
0.195.05

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